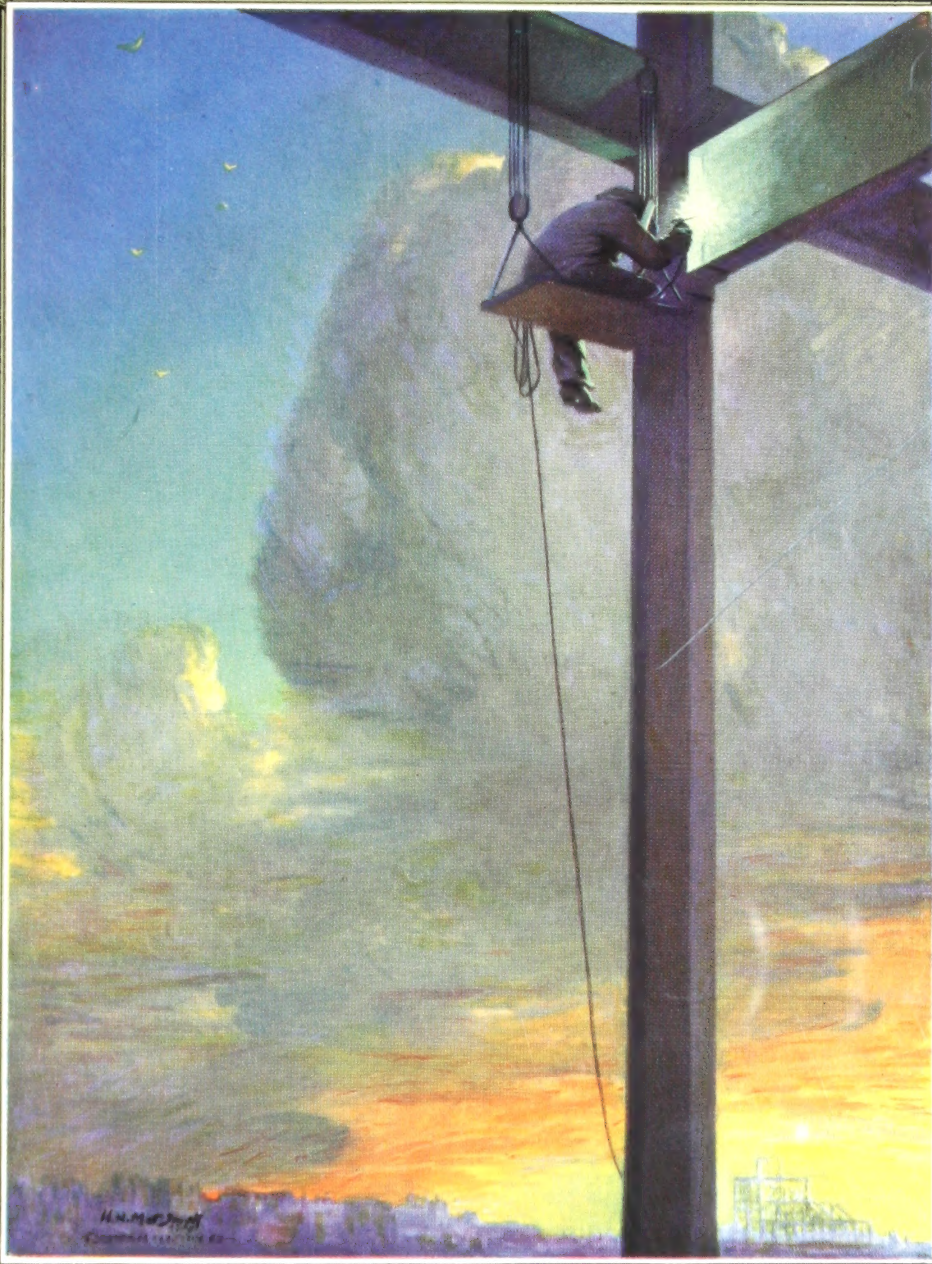

ARC WELDING STRUCTURAL STEEL

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ARC WELDING
STRUCTURAL STEEL

GENERAL ELECTRIC COMPANY
SCHENECTADY, N. Y.

July, 1930

GEA-1161

This publication is designed to present briefly, and in a form adapted for convenient reference, a number of facts which it is hoped will prove useful to those engaged in the design, fabrication, and erection of steel structures.

Arc Welding Structural Steel

THE electrical industry has recently made a very important contribution to the art of steel construction, which bids fair to be universally adopted as a marked aid in promoting efficiency, reliability, and economy in the fabrication and erection of steel structures. This contribution is the metallic arc weld. The development of structural joints used in building frame construction during the last half century is a matter of great interest. The first four stages of progress are outlined in the next paragraph; and now metallic arc welding is rapidly becoming recognized as the fifth step in the advancement of this type of design and construction.

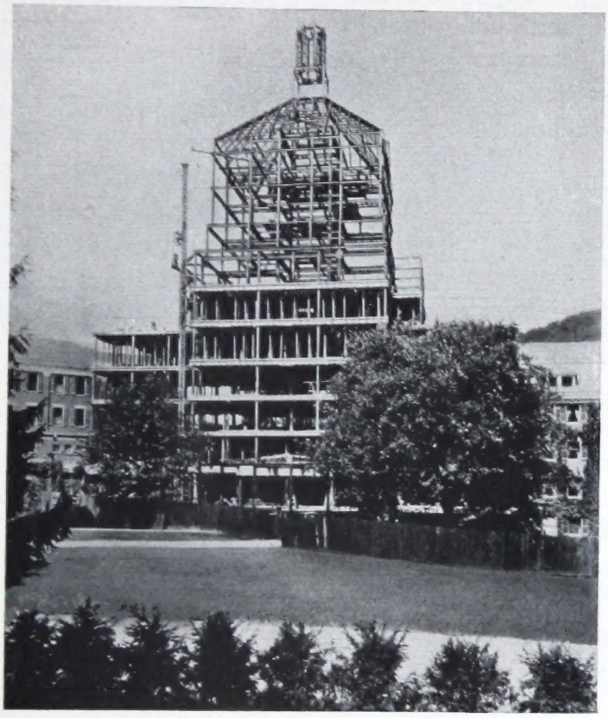


Fig. 1. Arc-welded Addition to the Homestead Hotel, Hot Springs, Va. Fabricated and Erected by the American Bridge Co.

Development of Structural Joints

The first step of advancement, the pin-connected joint, was the American standard for a long time prior to the development of the second step, or riveted joint. During the decade 1880 to 1890, there was a great deal of discussion among American engineers concerning the relative merits of the second step over the first step. By 1890, it was conceded that the riveted joint or truss was superior for most, but not for all, uses. About 1900, the riveted steel frame had definitely taken its place among the recognized standard methods of construction.

Following closely this step of advancement, came reënforced-concrete construction, which may be regarded as the third step. There were

many failures, but this type of building had come to stay, and led to the next step, the result of Considére's experiments on spirally reënforced columns. By 1910, large reënforced-concrete buildings, characterized by flat slabs and spirally reënforced columns, were being erected.

Prior to 1915, electric arc welding had been used for many different odd jobs. It was well known, but had not yet been applied to steel building construction. However, about this time welded steel structures made their appearance. Since then, many large factory buildings (the most worthy of note to date being perhaps the West Philadelphia Works of the General Electric Company), many bridges, hotels, office buildings, and other miscellaneous structures have been arc welded.

Some Advantages of Arc Welding

Saving in Steel

The reduction in the amount of steel required for arc-welded trusses in comparison with that required for riveted trusses is quite considerable. For example, Building 1 of the General Electric Company's plant at West Philadelphia was estimated as an 1100-ton riveted steel structure. When it was erected, with arc welding as a means of fabrication, only 987 tons of steel were used. A 58-ft., 6-in. welded truss weighed 5000 lb. as compared with 6800 lb., for a riveted truss, a net saving of 1800 lb., or 36 per cent of the arc-welded truss. A 78-ft. welded truss weighed 9200 lb. as against 13,200 lb. for a riveted truss, a saving of 4000 lb., or 43 per cent of the arc-welded truss. Naturally, these calculations indicate that engineers are justified in diligent study of the entire problem in order to obtain manifest economies in steel work.

Simplicity of Design

The elimination of gusset plates and of allowances for rivet holes is a certain indication of simpler design. This is emphasized and will be discussed later under the heading "Design" on page 6.

Labor Saving

Simplicity of design is attended by possibilities for savings in labor. There are no rivet holes to line up, no gusset plates to rivet in place before the members can be welded, and the welding of structural steel lends itself readily to the use of jigs. In shop work, where a great many trusses of the same design are to be fabricated, the jig will eliminate all layout and lining-up operations. In field work, a two-man crew consisting of a welder and his helper is far smaller than a riveting gang involving a heater boy, passer, riveter, holder-on, etc.

Noise

At the present time, silence is one of the main incentives for the use of arc welding instead of riveting. This is of particular advantage in congested areas, or in the neighborhood of hotels and residences. A short time ago, while the steel frame of a large office building was being riveted, the occupants of an adjoining office building were required to install telephone booths in each office in order to carry on telephone conversations. Only recently, a judge of the Supreme Court of Pennsylvania issued an order requiring a contractor to cease riveting on a nearby steel building frame so that it would be possible for him to conduct his court sessions. A few notable examples of the use of arc welding in locations where noise was undesirable are: the power house between the Chalfonte and Haddon Hall hotels at Atlantic City, N. J., the Homestead Hotel at Hot Springs, Va., and the General Electric Company's new factory building in West Philadelphia, Pa.

General Welding

In 1877, the first public demonstration of welding steel by the electric arc was shown in an experiment before the Franklin Institute at Philadelphia. Electric arc welding has progressed rapidly since these first experiments, and today there are three important divisions of the process. These are:

Metallic-arc welding

Carbon-arc welding

Atomic-hydrogen arc welding.

The last two of these three methods do not lend themselves readily to building work. When using the carbon arc, which is essentially an arc between a carbon pencil and the steel which is being welded, the operator must hold an electrode in one hand and a filler rod in the other. This is particularly true when making fillet welds, which at the present time are used almost exclusively for building construction. However, the carbon arc can be used for making small, short cuts and is sometimes desirable for this purpose in the field.

The atomic-hydrogen arc-welding process is analogous to the oxy-acetylene method and, for building purposes, has the same disadvantages as the carbon arc; i.e., the filler rod must be held in one hand and the electrode holder in the other. This process has some outstanding advantages as compared with other methods of welding; for example, the actual heat of the flame. The atomic-hydrogen flame is approximately 4000 degrees C. as compared with 3200 degrees C. of the oxy-acetylene flame and 3600 degrees C. of the electric arc.

Metallic-arc welding is most applicable to structural welds. It can be divided under two headings, automatic welding and hand welding. Each of these has its place in the making of a structure. Columns, girders, beams, etc., are fabricated in the shops by automatic arc welding, as shown in Fig. 8 on page 9. Hand welding is used for the assembly of girders, trusses, etc., in the shop and field.

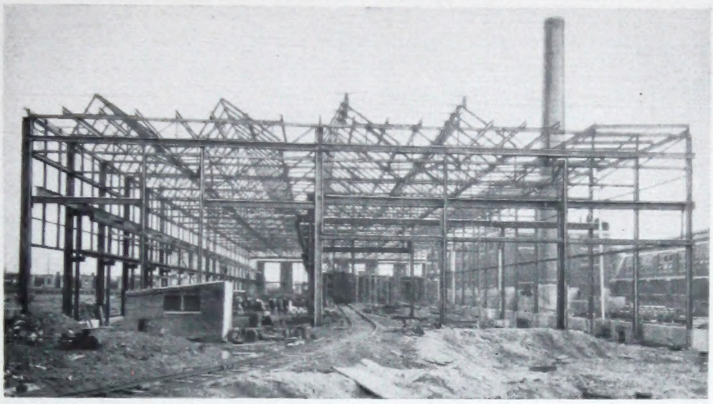


Fig. 2. Completely Arc-welded Building at the Philadelphia Works of the General Electric Company

Preliminary Investigations

Before applying the metallic arc scientifically to steel structures, much experimental work was necessary. General Electric, a pioneer in the use of arc welding and the largest user in the world of arc welding in its own plants, gladly assumed this obligation and the major portion of the results obtained are included in this publication. The testing of welds in building construction can be classified as follows:

- Longitudinal shear on fillets
- Transverse shear on fillets, involving also simultaneous tensile or compressive stresses on some planes within the fillets
- Tensile and compressive strength of welds in butt joints.

Longitudinal shearing values of fillets are sufficiently well established for safe design of

ordinary connections. Also, the tensile and compressive strength of welds in butt joints has been given a great deal of attention. The former are useful when designing joints in pipes and cylindrical tanks, while the latter are of value in planning joints, such as butt splices, in some compression members—for example, top-chord splices. Many tests are at present in process which add daily to the already considerable amount of available data on this subject.

6, the locations of the welds on the specimens are shown.

From these tests, it may be concluded that the proper design value to be used is 2000 lb. per linear inch for $\frac{1}{4}$-in. welds; 2500 lb. per linear inch for $\frac{5}{16}$-in. welds; 3000 lb. per linear inch for $\frac{3}{8}$-in. welds; 4000 lb. per linear inch for $\frac{1}{2}$-in. welds; 5000 lb. per linear inch for $\frac{5}{8}$-in. welds; and 6000 lb. per linear inch for $\frac{3}{4}$-in. welds.

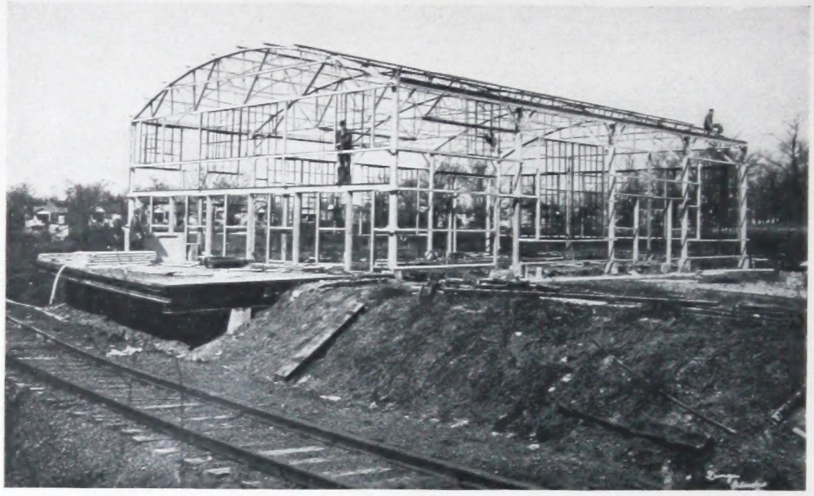


Fig. 3. Another All-arc-welded Factory Building

Tests

There are now available results of 242 tests of metallic arc welds, which indicate the uniformity of strength and reliability of the joints. Fig. 4, 5, and 6 show the results of these tests.

There are several facts to be observed from these diagrams. Each point of Fig. 4 represents an average of three tested specimens; and points of the same size fillets, but differing in length, are connected together. It should be noted that, for the larger fillets, a minimum length should be agreed on if an average value of ultimate shearing strength is to be used.

Fig. 5 shows the results of the tests of several samples when the welds are not opposite, while Fig. 6 shows the shearing strengths of the compression tests. It will be noted that on each of the three illustrations, Fig. 4, 5, and

Fig. 7 and 11 show several tested specimens used in compiling the test data. Observation of these specimens shows that the various types of welds which will be encountered in good structural design are represented and also that the bars of the test specimens were sufficiently large so that the stresses in the bars were well within their elastic limit at the time the welds failed. In other words, the welds and not the bars were tested.

Design

Welded steel structures include several distinct parts, such as trusses, columns, beams, and girders. Trusses require a detailed analysis in order to determine the stresses in the various members produced by the applied loads. Columns require tabulation of all of the loads

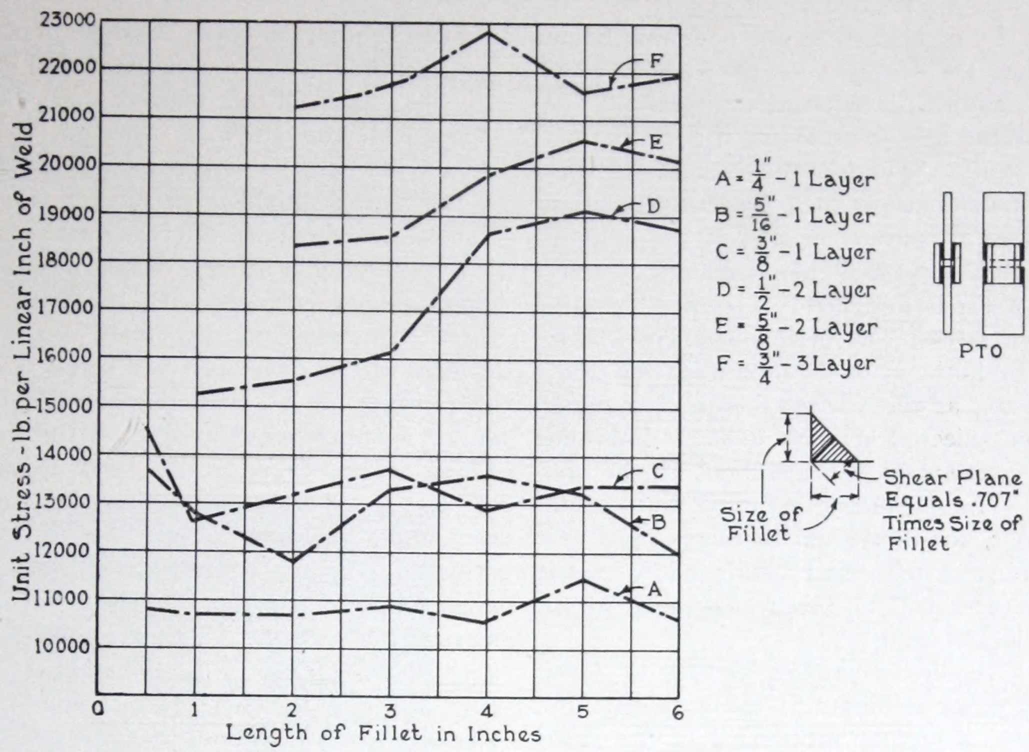


Fig. 4. Curves Showing Unit Stress of "PTO" Specimens

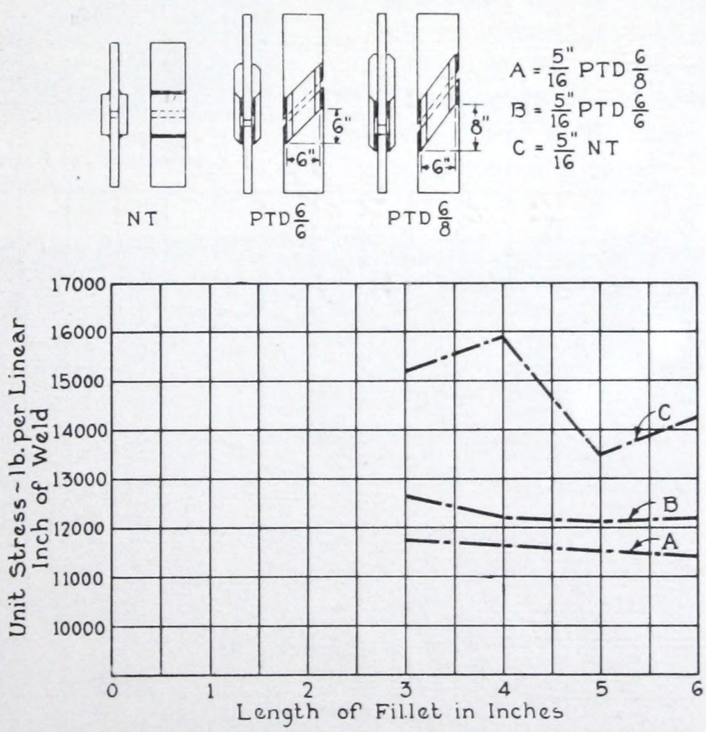


Fig. 5. Curves Showing Unit Stress of "NT" and "PTD" Specimens

Legend:	P = Parallel	O = Opposite	D = Diagonal
	T = Tension	N = Normal	

tributary to, or applied to the columns. Beams and girders are designed for the load to which they are subjected, whether uniform or concentrated; but also, in preparing the calculations, the reactions at the ends of the beams and girders are determined in order to design the connections from beams to girders or girders to columns. Occasionally, structural members and connections must also be designed for combined bending and direct stress. The design problems, therefore, are preliminary to the determination of the forces acting at the various joints. The design data thus collected are also used to determine the arrangement of the members at the connections. The problems involved for various connections, for individual members, and also the assembly of individual members into trusses and structures will be briefly discussed in the paragraphs following.

Trusses require a greater variety of joints than any other elements in the structure. For this reason, a greater proportion of the study is given to joints in them. Analysis of such joints is a reminder that the fundamental principles are the same for all trusses, and these principles will, therefore, be restated. Truss lines should intersect at a point, and loads applied to the trusses should be applied at these points. The center of gravity of truss members and the center of gravity of the welds applied at the joints should coincide with the truss lines. If these principles are observed, no secondary stresses will be developed as a result of eccentric connections or because the center of gravity of the member is not located on the truss lines. These principles naturally tend to encourage the use of symmetrical sections for truss members, but, since it is not always economical or practical to use symmetrical sections, methods must be developed for utilizing nonsymmetrical sections. To accomplish this, joint details have been developed for nonsymmetrical members, such as angles, which are frequently used. Angles would require the use of eccentric

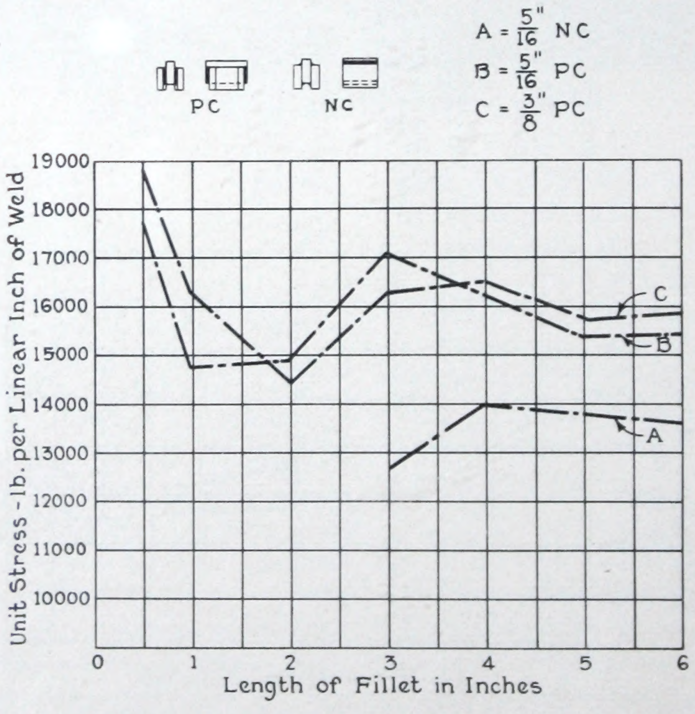


Fig. 6. Curves Showing Unit Stress of "PC" and "NC" Specimens

Legend: P = Parallel; C = Compression; N = Normal

joints, except for the fact that it is possible to proportion the welding used for connecting angles to other members in such a manner that the center of gravity of the amount of welding used coincides with the center of gravity of the member and the truss lines. The design of joints for trusses involves, therefore, the use of symmetrical connections, occasioned by the use of tees and channels, and the design of nonsymmetrical connections, occasioned by the use of angles. Data and examples are given of various connections used in the design of arc-welded trusses.

Fig. 9, 10, 12, and 14, with their accompanying tables of data, as well as Fig. 15 and 16 illustrate the use of members found in arc-welded steel structures. Fig. 9 illustrates single-angle connections. The dimensions and properties of the section of an angle determine the amount of welding to be placed on either edge of the angle. Moments about the center of gravity of the angle produced by the amount of welding used are equal so that no unbalanced moment will be transmitted to the member or unequal stresses applied to the welds. The derivation of the formula and examples given in Fig. 9 illustrate this important problem. Standard single-channel tension connections are given in Fig. 10 and standard single-tee tension connections are given in Fig. 12, but as both of these members are symmetrical about their center lines, the number of linear inches of weld is made equal on both edges of the members. Fig. 14 illustrates the connections used for typical 2-angle "Z" struts. The examples given in Fig. 14, coupled with those given in Fig. 15, illustrate the methods used to determine the amount and position of the weld. Fig. 16 shows typical connections of single-angle members to a chord composed of one-half of an "H" beam with all the calculations prepared in detail.

Fig. 7. Tension Test Specimens of Fillet Welds

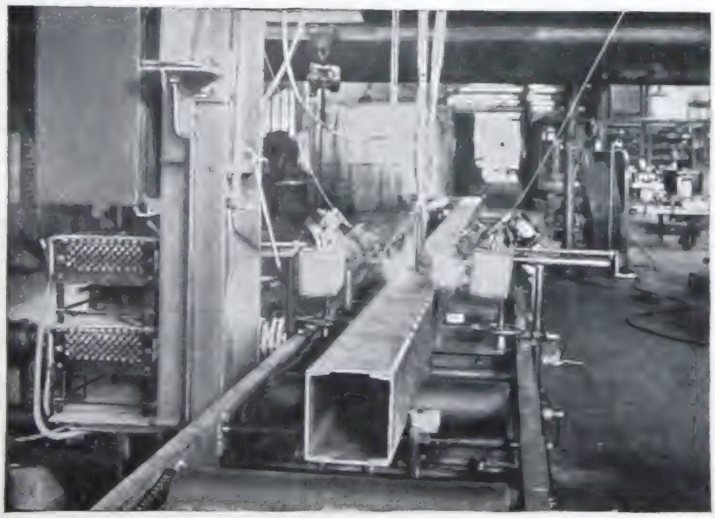
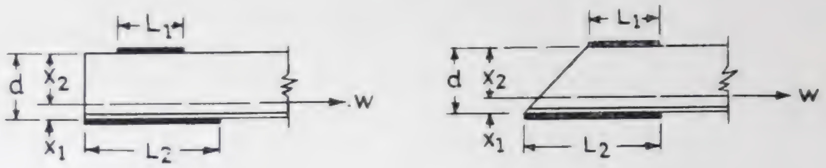


Fig. 8. Automatic Arc Welder Fabricating Box Columns in the Shop

Development of arc-welded trusses encouraged the use of new types of members and, also, the use of members heretofore not extensively used for trusses. Analysis of chord sections indicated that, if gusset plates were to be eliminated, it would be necessary to have wide, flat surfaces parallel to the plane of the truss on which the web members could be welded. Further study indicated that "H" beams and large tees, obtained by splitting large beams

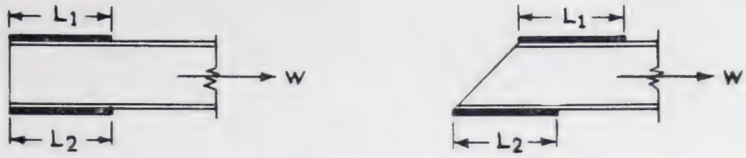


W = Load on Angle. S = Unit Stress per lin. in. Weld.
L = Length of Weld in Inches. $W/S = L$ $L = L_1 + L_2$
$L_1 d = L x_1$ $L_1 = L x_1/d$ $L_2 d = L x_2$ $L_2 = L x_2/d$

EXAMPLES:
Strut $4 \times 4 \times \frac{3}{8}$ L $\times$ 7 Ft.-0 In. $W = 31600$ $S = 3000$
$L = W/S = 10.5$ In. $L_1 = 10.5 \times .285 = 3.0$ In. $L_2 = 10.5 \times .715 = 7.5$ In. Detail $L_1 = 3$ In. $L_2 = 7\frac{1}{2}$ In.
Tension $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ L $\times$ 18000 Lb. sq. in. = 21420 Lb.
From Table $L_1 = 3.1$ In. $L_2 = 7.6$ In. Detail $L_1 = 3\frac{1}{4}$ In. $L_2 = 7\frac{3}{4}$ In.

Size Angle	d	x_1	x_2	Weld	S	x_1/d	x_2/d	Area	A × 18000	L	L_1	L_2
2×2×1/4	2	.59	1.41	1/4	2000	.295	.705	.94	16920	8.5	2.5	6.0
5/16	2	.61	1.39	5/16	2500	.305	.695	1.15	20700	8.3	2.5	5.8
3/8	2	.64	1.36	3/8	3000	.320	.680	1.36	24480	8.2	2.6	5.6
2½×2½×1/4	2½	.72	1.78	1/4	2000	.288	.712	1.19	21420	10.7	3.1	7.6
5/16	2½	.74	1.76	5/16	2500	.296	.704	1.47	26460	10.6	3.1	7.5
3/8	2½	.76	1.74	3/8	3000	.304	.696	1.73	31140	10.4	3.2	7.2
3×3×1/4	3	.84	2.16	1/4	2000	.280	.720	1.44	25920	13.0	3.6	9.4
5/16	3	.87	2.13	5/16	2500	.290	.710	1.78	32040	12.8	3.7	9.1
3/8	3	.89	2.11	3/8	3000	.297	.703	2.11	37980	12.7	3.8	8.9
3½×3½×5/16	3½	.99	2.51	5/16	2500	.283	.717	2.09	37620	15.0	4.2	10.8
3/8	3½	1.01	2.49	3/8	3000	.289	.711	2.48	44640	14.9	4.3	10.6
7/16	3½	1.04	2.46	3/8	3000	.297	.703	2.87	51660	17.2	5.1	12.1
4×4×3/8	4	1.14	2.86	3/8	3000	.285	.715	2.86	51480	17.2	4.9	12.3
7/16	4	1.16	2.84	3/8	3000	.290	.710	3.31	59580	19.9	5.8	14.1
1/2	4	1.18	2.82	3/8	3000	.295	.705	3.75	67500	22.5	6.7	15.9
6×6×3/8	6	1.64	4.36	3/8	3000	.273	.727	4.36	78480	26.2	7.1	19.1
1/2	6	1.68	4.32	3/8	3000	.280	.720	5.75	103500	34.5	9.7	24.8
5/8	6	1.73	4.27	3/8	3000	.288	.711	7.11	127980	42.7	12.3	30.4
4×3×3/8	4	1.28	2.72	3/8	3000	.320	.680	2.48	44640	14.9	4.8	10.1
3/8	3	.78	2.22	3/8	3000	.260	.740	2.48	44640	14.9	3.9	11.0
7/16	4	1.30	2.70	3/8	3000	.325	.675	2.87	51660	17.2	5.6	11.6
4×3×7/16	3	.80	2.20	3/8	3000	.267	.733	2.87	51660	17.2	4.6	12.6
1/2	4	1.33	2.67	3/8	3000	.332	.668	3.25	58500	19.5	6.5	13.0
1/2	3	.83	2.17	3/8	3000	.277	.723	3.25	58500	19.5	5.4	14.1
5×3½×3/8	5	1.61	3.39	3/8	3000	.322	.678	3.05	54900	18.3	5.9	12.4
3/8	3½	.86	2.64	3/8	3000	.246	.754	3.05	54900	18.3	4.5	13.8
7/16	5	1.63	3.37	3/8	3000	.326	.674	3.53	63540	21.2	6.9	14.3
5×3½×7/16	3½	.88	2.62	3/8	3000	.251	.749	3.53	63540	21.2	5.3	15.9
1/2	5	1.66	3.34	3/8	3000	.332	.668	4.00	72000	24.0	8.0	16.0
1/2	3½	.91	2.59	3/8	3000	.260	.740	4.00	72000	24.0	6.2	17.8
6×4×3/8	6	1.94	4.06	3/8	3000	.323	.677	3.61	64980	21.7	7.0	14.7
3/8	4	.94	3.06	3/8	3000	.235	.765	3.61	64980	21.7	5.1	16.6
1/2	6	1.99	4.01	3/8	3000	.332	.668	4.75	85500	28.5	9.5	19.0
6×4×1/2	4	.99	3.01	3/8	3000	.248	.753	4.75	85500	28.5	7.1	21.4
5/8	6	2.03	3.97	3/8	3000	.338	.662	5.86	105480	35.2	11.9	23.3
5/8	4	1.03	2.97	3/8	3000	.258	.743	5.86	105480	35.2	9.1	26.1

Fig. 9. Standard Single-angle Connections



W = Load on Channel. S = Unit Stress per lin. in. Weld.
L = Length of Weld in Inches. $W/S = L$ $L_1 = L_2 = L/2$

EXAMPLES:

3-in. Channel 4.1 Lb. W = 21400 S = 3000 $L = W/S = 7.1$ In. $L_1 = L_2 = 3.6$ In.
Detail $L_1 = L_2 = 3\frac{3}{4}$ In.

5-in. Channel 9.0 Lb. W = 47300 S = 3000 $L = W/S = 15.8$ In. $L_1 = L_2 = 7.9$ In.
Detail $L_1 = L_2 = 8$ In.

Size Channel	Weight	Area	Area ×18000	Weld.	S	L	$L_1 = L_2$
3	4.1	1.19	21400	3/8	3000	7.1	3.6
	5.0	1.46	26300	3/8	3000	8.8	4.4
	6.0	1.75	31500	3/8	3000	10.5	5.3
4	5.4	1.56	28100	3/8	3000	9.4	4.7
	6.25	1.82	32800	3/8	3000	10.9	5.5
	7.25	2.12	38200	3/8	3000	12.7	6.4
5	6.7	1.95	35100	3/8	3000	11.7	5.9
	9.0	2.63	47300	3/8	3000	15.8	7.9
	11.5	3.36	60500	3/8	3000	20.2	10.1
6	8.2	2.39	43000	3/8	3000	14.3	7.2
	10.5	3.07	55300	3/8	3000	18.4	9.2
6	13.0	3.81	68600	3/8	3000	22.8	11.4
	15.5	4.54	81760	3/8	3000	27.2	13.6
7	9.8	2.85	51300	3/8	3000	17.1	8.6
	12.25	3.58	64400	3/8	3000	21.5	10.8
	14.75	4.32	77800	3/8	3000	25.9	13.0
7	17.25	5.05	90900	3/8	3000	30.3	15.2
	19.75	5.79	104200	3/8	3000	34.7	17.4
8	11.5	3.36	60500	3/8	3000	20.2	10.1
	13.75	4.02	72400	3/8	3000	24.1	12.1
	16.25	4.76	85700	3/8	3000	28.6	14.3
8	18.75	5.49	98800	3/8	3000	32.9	16.5
	21.25	6.23	112100	3/8	3000	37.4	18.7

Fig. 10. Standard Single-channel Tension Connections

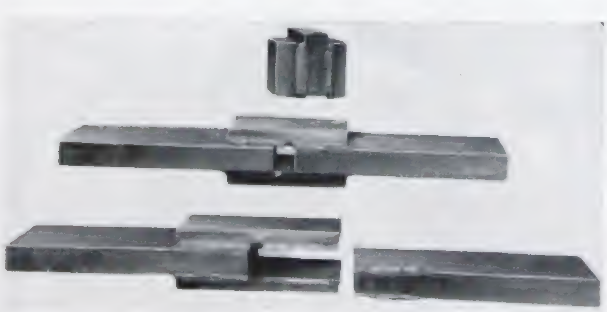


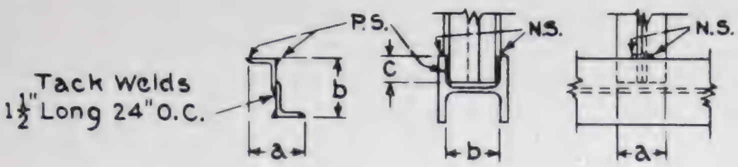
Fig. 11. Compression and Tension Test Specimens After Failure

W = Load on Tee. S = Unit Stress per lin. in. Weld.
L = Length of Weld in inches. $W/S = L$ $L_1 = L_2 = L/2$
EXAMPLES:
2×2 Tee 4.3 Lb. W = 22700 S = 2500 L = W/S = 9.1 In. $L_1 = L_2 = 4.6$ In.
Detail $L_1 = L_2 = 4\frac{3}{4}$ In.
4×3 Tee 9.2 Lb. W = 48200 S = 3000 L = W/S = 16.1 In. $L_1 = L_2 = 8.1$ In.
Detail $L_1 = L_2 = 8\frac{1}{4}$ In.

Size Tee	Weight	Area	Area ×18000	Weld	S	L	$L_1 = L_2$
2×2	3.56	1.05	18900	1/4	2000	9.5	4.8
2×2	4.3	1.26	22700	5/16	2500	9.1	4.6
2¼×2¼	4.1	1.19	21400	1/4	2000	10.7	5.4
2¼×2¼	4.9	1.43	25700	5/16	2500	10.3	5.2
2½×2½	5.5	1.60	28800	5/16	2500	11.5	5.8
2½×2½	6.4	1.87	33700	3/8	3000	11.2	5.6
3×3	6.7	1.95	35100	5/16	2500	14.0	7.0
3×3	7.8	2.27	40900	3/8	3000	13.6	6.8
4×4	10.5	3.09	55600	3/8	3000	18.5	9.3
4×4	13.5	3.97	71500	3/8	3000	23.8	11.9
6½×6½	19.8	5.80	104400	3/8	3000	34.8	17.4
2½×3	6.1	1.77	31900	5/16	2500	12.8	6.4
3×2½	6.1	1.77	31900	5/16	2500	12.8	6.4
4×2½	7.2	2.12	38200	5/16	2500	15.3	7.7
4×2½	8.5	2.48	44600	3/8	3000	14.9	7.5
4×3	7.8	2.29	41200	5/16	2500	16.5	8.3
4×3	9.2	2.68	48200	3/8	3000	16.1	8.1
4×4½	11.2	3.29	59200	3/8	3000	19.7	9.9
4×4½	14.4	4.23	76100	3/8	3000	25.4	12.7
4×5	11.9	3.49	62800	3/8	3000	20.9	10.5
4×5	15.3	4.50	81000	3/8	3000	27.0	13.5
5×3	11.5	3.37	60700	3/8	3000	20.2	10.1

Fig. 12. Standard Single-tee Tension Connections

Fig. 13. Detail at Apex of Arc-welded Truss



W = Load on Strut. S = Unit Stress per lin. in. Weld. L = Length of Weld in inches. F = Allowable Unit Stress on Strut = $16000-70 \times l/r$. L = W/S. N.S. = Normal Shear Weld. P.S. = Parallel Shear Weld. c = Length Available for P.S.

EXAMPLES:

2 Angles $4 \times 3 \times \frac{5}{16}$ a = 6 In. b = 7.2 In. $\frac{5}{16}$ In. Weld. S = 2500 r = 1.2
l = 10 Ft. = 120 In. F = $16000 - 70 \times 120/1.2 = 9000$. Area = $2 \times 2.09 = 4.18$
W = $4.18 \times 9000 = 37620$ L = $37620 \div 2500 = 15.05$ In. c = $3\frac{3}{8}$ In.
L $- 4 \times 3\frac{3}{8} = 15.05 - 13.5 = 1.55$ $1.55/2 = .78$ Detail N.S. = 1 In.

2 Angles $4 \times 4 \times \frac{3}{8}$ a = 8 In. b = 7.2 In. $\frac{3}{8}$ In. Weld. S = 3000 r = 1.69 l = 10 Ft. = 120 In.
F = $16000 - 70 \times 120/1.69 = 11030$. Area = $2 \times 2.86 = 5.72$. W = $5.72 \times 11030 = 63200$
L = $63200 \div 3000 = 21.07$ In. c = $3\frac{3}{8}$ In. L $- 4 \times 3\frac{3}{8} = 21.07 - 13.5 = 7.57$ $7.57/2 = 3.79$
Detail N.S. = 4 In.

Fig. 14. Typical Two-angle Z-strut Connections

with the aid of the carbon arc, would provide suitable chord sections and the necessary surfaces to which the web members could be welded. The "H"-beam section, placed with its web horizontally, as illustrated in Fig. 17, provided a chord exceedingly stiff or strong in the horizontal plane. This reduced considerably the amount of horizontal bracing required for trusses of a long span. Beams split in two as shown in Fig. 18 provided tees of capacity equal to the pairs of angles formerly used in riveted design, and also provided the wide, flat surfaces to which web members could be welded.

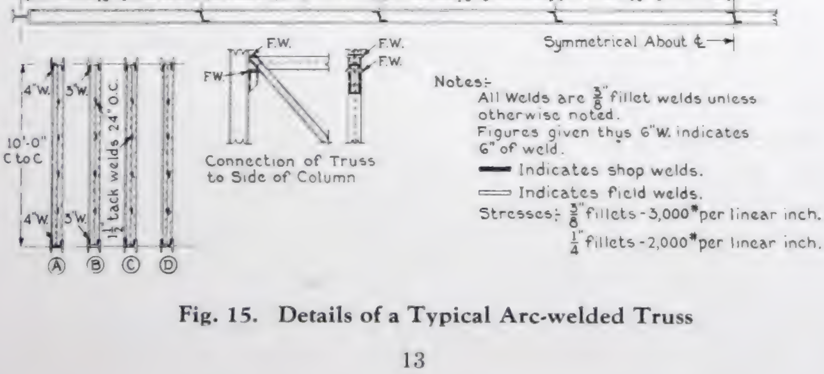


Fig. 15. Details of a Typical Arc-welded Truss

Even the web members of trusses were modified in design to suit welded joints. Channels are extensively used for tension members, since it is possible to use ⅜-in. welds on both web edges of the channels and thus develop welds with a capacity of 3000 lb. per linear inch of weld, resulting in short connections. Channels are used in the design of joints, and the amount of welding required is frequently so long that it is impossible to place the welds directly opposite each other. If necessary, in such cases, welds can be placed on both edges of the channels across the full width of the chord member. This indicates that welds placed in such positions are practically equal in strength to those placed directly opposite each other. The same unit stresses, therefore, are used for such welds. Tees are also suitable for use as web members, and their connections are designed similarly to those given for the connections for channels. Angles are used, with preference given to angles with metal ⅜ in., or more, thick so that ⅜-in. welds can be made. Where it is economical and practical to use angles with metal ¼ in. or $\frac{5}{16}$ in. thick, ¼-in. or $\frac{5}{16}$-in. welds are used with the welds designed on the basis of 2000 lb. per linear inch for ¼-in. welds and 2500 lb. per linear inch for $\frac{5}{16}$-in. welds. The design of joints for compression or tension web members consisting of single or paired angles is given in detail in Fig. 9. When "H" beams are used for chord sections, either an "I" beam or a new form of strut, known as the "Z" strut, is used. The "Z" strut, illustrated in Fig. 14, consists of two angles so arranged that adjacent legs are welded together to convert the two angles into a single member. The outer legs of the angles are welded to the inner surfaces of the flanges of the "H" beams. Unfortunately, the connection of a "Z" strut to the inner surface of the "H"-beam chord section is somewhat complicated by the fact that the plane surface of the "H" beam does not extend all the way to the web because of the fillet, so necessary between the web and the flange. However, as a rule, sufficient welding can be applied to the four edges of the "Z" strut in contact with the inner surfaces of the "H" beam. If this amount of welding is insufficient, additional welding can be applied between the "Z" strut and the edges of the chords. It is also possible to place a plate between the end of

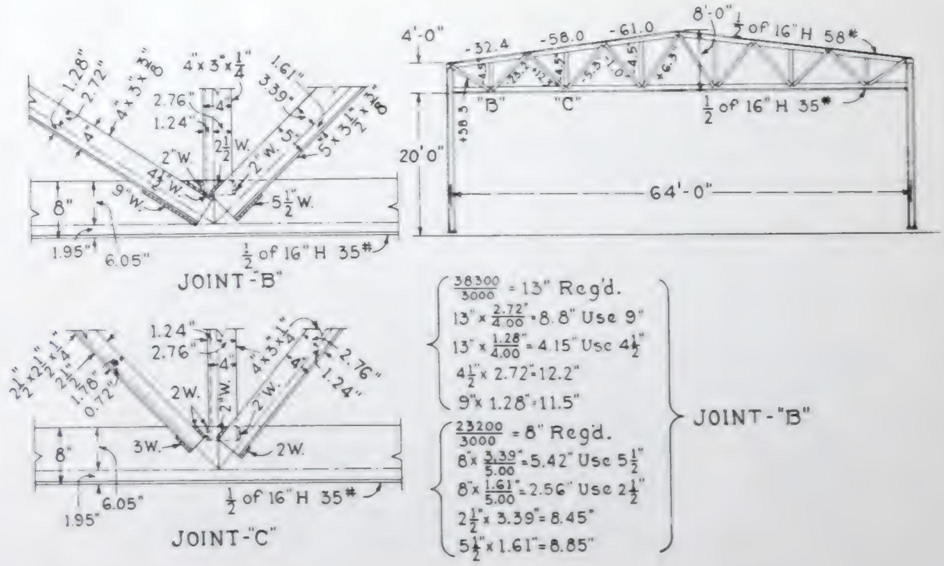


Fig. 16. Details of Another Type of Arc-welded Truss

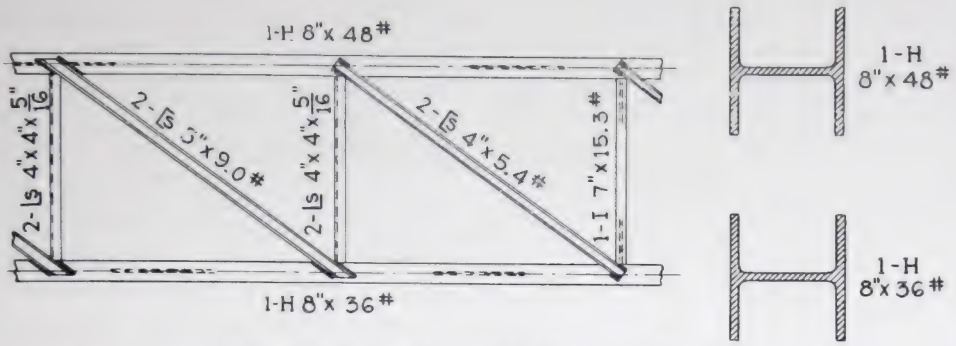


Fig. 17. Typical Arc-welded Pratt-type Truss

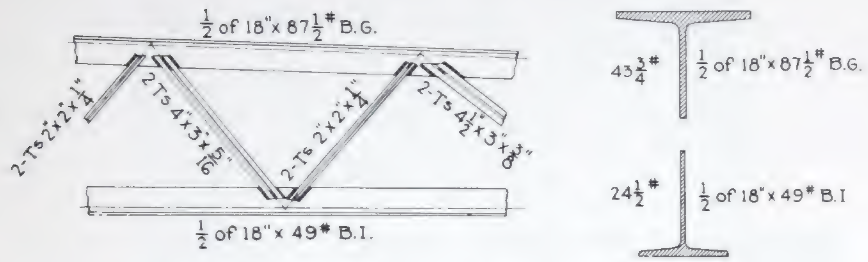


Fig. 18. Typical Arc-welded Warren-type Truss

the "Z" strut and the web of the chord, but this additional provision is required only in the case of exceptionally heavy trusses.

The connections of beams to girders and of girders to columns involve two types of connections, known as bracket and web connections. Present experience indicates that bracket connections are cheaper than web connections, because the bracket connections can be fabricated in the shop and the brackets can contain all the holes required for fastening the beams to the girders or the girders to the columns. The fundamental principle in the design of welded connections is that all punching should be performed, as far as is possible, in the clip angles or bracket details and that punching should be avoided in all heavy members. If brackets are well designed and attached to the girders or columns in the shop, field welding is minimized. The design of welded brackets has not reached the point where it is possible to call any design either typical or standard. This detail usually consists of a clip angle, as illustrated in Fig. 19 with the horizontal leg of the angle of a width sufficient to permit punching $\frac{13}{16}$-in. diameter holes for $\frac{3}{4}$-in. bolts in order to support and hold the girder or beam securely in position. The vertical leg of the angle should be of sufficient width to provide on each end of the angle welding edges which are of a size sufficient to support the load to be transmitted from beam to girder or girder to column. As an example, take a beam with a reaction of 35,000 lb. This weight, divided by 3000 (lb. per linear inch for a $\frac{3}{8}$-in. weld) would require $11\frac{2}{3}$ (or 12) linear inches of weld. If the weld should be applied to the ends of the clip angle, the leg of the clip angle must be 6 inches wide to permit the proper amount of welding at each end. If a heavier reaction must be provided for, a similar procedure is required, except that it is frequently necessary to place the angles in a vertical position and

then to place a plate on top of the angles to provide the seat for the beams or girders.

The connection of columns to base plates or foundations is illustrated in Fig. 20, which shows an equal leg angle welded to a steel column at its foot. The use of an angle or a bent plate in "U" form is quite general. The method of design is simple. The stress in the anchor bolt is determined and divided by 3000 (lb. per linear inch) in order to obtain the number of linear inches of welding required. This result is then divided by 2 to obtain the length of the angle or "U" plate, as two lines of welding are applied to the edges of the equal leg angle or "U"-shaped plate. If horizontal forces are to be applied to the column, the column can be field welded to the base plate and the base plate incased in concrete and properly secured to the foundation by reënforcing steel, all as indicated in the illustration.

The design of joints for welded steel structures is in process of development. The designs illustrated in this paper are some of those which have been developed up to the present time. There is opportunity for much further study and improvement, particularly with reference to brackets used for supporting beams and girders and also those used for wind bracing. There are many other forms of joints requiring considerable study, many tests and additional new developments in both shop and field to be made. As new joints and connections are developed, photographs, drawings, and calculations should be procured and filed whenever possible, inasmuch as they will furnish valuable reference data. The development of welded steel structures now depends largely upon the development of the details of connections and joints.

Fig. 19. Details of an Arc-welded Beam Seat

Qualifications of Welders

There is a tendency to exaggerate the importance of the human factor in arc welding. It is certainly desirable to have good welding operators, but over-zealousness in this particular is not wise, since it tends to create a scepticism regarding the safety of welds. This increases the cost unnecessarily, because it is likely to burden welders with rules not applied to other workmen, such as riveters and concrete workers, and thus creates unnecessary municipal bureaus.

On important buildings or bridge work, the employment of welders merely upon their claims of experience in arc welding is not sufficient. While their claims may be perfectly correct and made in good faith, nevertheless it is true that a welder may be capable of

Fig. 20. Details of Arc-welded Column Anchors

Fig. 21. Arc-welded Overhead Railroad Crossing, Adopted for Safety

obtaining excellent results in one class of work and yet be unable to perform well the particular work at hand. Each man should be tested by at least one of the following three methods.

Butt Welds

Welders should be required to weld four sample butt joints, each made of two pieces ½ in. by 9 in. by 12 in. to form a piece about 12 in. by 18 in. prepared with a double "V" later described. Two of these sample plates should be welded in a flat position and two in a vertical position with the joint located vertically. Each sample plate should be machined to reduce the joints to the thickness of the base metal. From each sample plate, the standard 2-in. tensile test specimens should be made and tested in tension. The average tensile strength of each group of two sample plates should not be less than 45,000 lb. per sq. in., and the tensile strength of the lowest in each group should not be less than 40,000 lb. per sq. in.

Steel for double-"V" joints should be beveled not less than 37½ degrees on each side of each piece to form open angles of not less than 75 degrees.

Lap Welds

Welders should be required to weld four sample lap joints each made of ½-in. by 6-in. by 8-in. plates clamped one on the other with the 8-in. edges lined up, but the 6-in. edges offset ½ in. A full ½-in. fillet weld should then be made along one 6-in. edge in accordance with specifications described later for lap joints. Upon completion of the weld and after cooling, the specimens should be torn apart and broken by wedging at the unwelded 6-in. edges and the fractured metal of the specimens broken through the weld to show metal—bright, dense, even-textured, either crystalline or fibrous, irregularly torn, and void of iridescent colors; good fusion of weld and base metal; and good penetration into the right angle corner of the fillet.

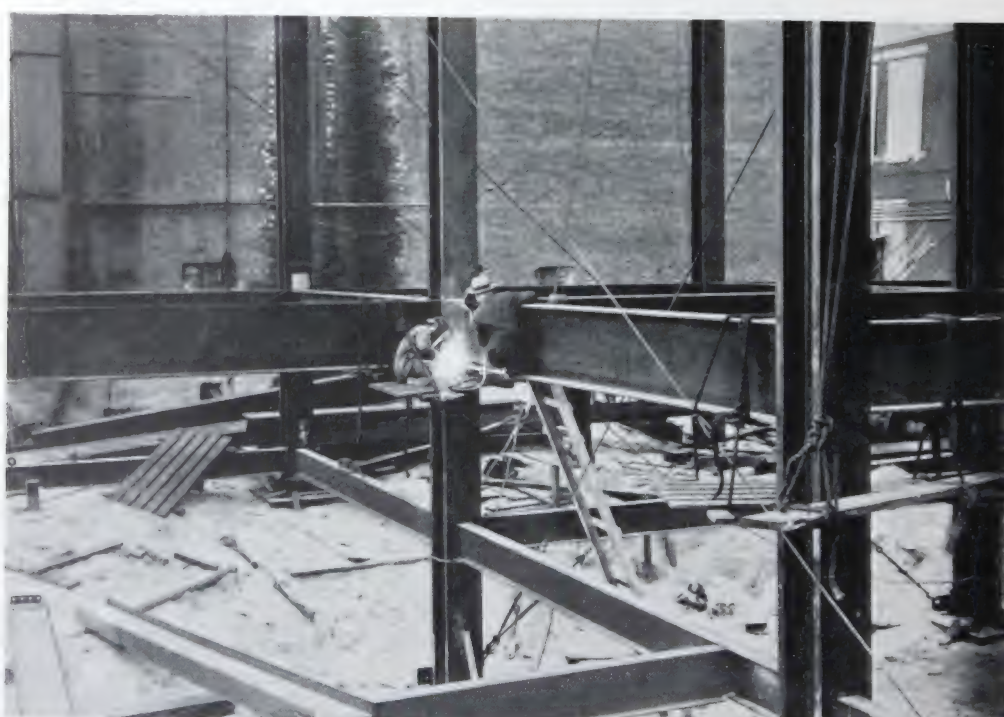


Fig. 22. Welder at Work on Power House of Chalfonte and Haddon Hall Hotels, Atlantic City, N. J.

All lap joints of ½-in. to $\frac{11}{16}$-in. steel shall be fillet-welded in two layers. The contour of the cross section of the fillets shall be as near as is practical to a triangle with equal sides and with minimum size not less than the prescribed dimensions of cross section of fillets.

Fillet Welds

Welders should be required to weld three sample test specimens of size shown in Fig. 23, each specimen to consist of two main plates 4 in. by 1 in. by 12 in., placed in line with their inner ends separated ½ inch, making the over-all length 24½ inches. These plates are to be connected by two splice plates, each 3 in. by ¾ in. by 6½ in., symmetrically clamped to the main plates, one above and the other below the two main plates, and each welded in a flat position to each main plate by two 2½-in. by ⅜-in. triangular fillets with their craters filled; each of these fillets to be started at outer ends of the 3-in. by ¾-in. by 6½-in. spliced plate and continued 2½ inches along the 6½-in. dimension toward the center, thus leaving a 1½-in. space between the inner ends of the fillets. The specimen should then be inverted and the four remaining fillets also deposited in the flat position. When depositing the weld metal, craters should be filled to full cross section of the fillet. Electrodes of $\frac{3}{16}$-in. diameter should be used. These specimens are to be tested in tension to ascertain the longitudinal shearing value of the ⅜-in. fillet welds and the fractured specimens should be examined for: penetration into, and fusion with, base metals; freedom from gas holes and from slag inclusions in fillets. The three specimens should show at least an average ultimate longitudinal stress of 42,000 lb. per sq. in., and the lowest in the group should show at least 38,000 lb. per sq. in., of minimum section of fillet.

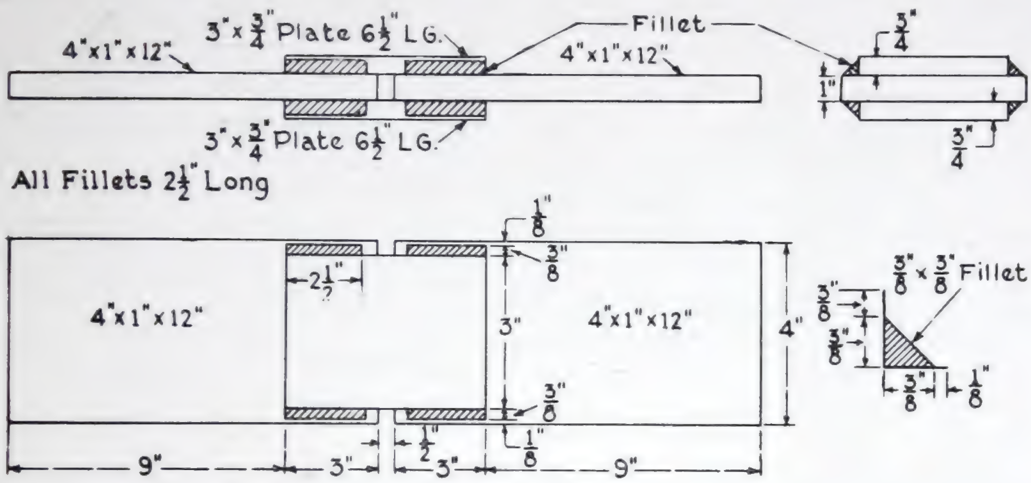


Fig. 23. Test Specimen for Qualification of a Welder

Of these test methods, that to determine the longitudinal shear value, Fig. 23, is the best, but most expensive, for ordinary building construction involving trusses, beams, girders, and columns. The test specimen shown in Fig. 23 is designed in such a manner that, when the 10 inches of fillet reach their ultimate shearing strength, the unit tension in main and spliced plates will not materially exceed the elastic limit. In fairness to operators, they should be given an opportunity, before making test specimens, of familiarizing themselves with the type of electrode and current density to be employed. It is also desirable to test welders in this manner from time to time during the progress of the work.

The authorities of one city have recently suggested that all welders employed be licensed by the Municipal Bureau of Building Inspection. Such action seems unnecessary. However, it is necessary that the engineer and architect in charge of a welded steel building frame have in their employ a competent inspector, who qualifies the welders by any, or all, of the methods described above. If municipal regulation of any kind is necessary, it would seem better to have the contractor for the steel submit evidence that the welders he purposes to employ on the building in question have met these or similar qualification tests.

Inspection

As in riveted steel and in reënforced concrete construction, the conservative engineer will provide for inspection of all welded work both in the shop and in the field. It is the duty of the inspector to see that the sizes and lengths of steel sections are correct, that welders are well qualified, that the proper current and electrodes are used, that the current values are changed to suit the various sizes of fillets and thicknesses of steel, that the sizes and lengths of fillets are deposited to conform with those specified on the drawings, and that all parts conform to drawings as to size and position.

A good inspector can learn much by visual inspection of a fillet weld. This has been proved by tests which General Electric has made. Rounded edges denote lack of penetration of the fillet into the parent metal, i.e., into the parts being welded. One short and one long side of a triangular fillet indicates that the electrode wire has been held at an incorrect angle while welding. A crater at any point in the fillet, other than the end, is evidence that the arc has been broken and the fillet not continuously laid. Numerous gas holes on the surface of the fillet indicate too long an arc and lack of penetration. A current shown by the ammeter on the welding machine to be too great for thin plates, or too light for thick ones, is undesirable and the inspector should see that the welders adjust their machines to obtain the current suitable for the thickness of the material being welded.

When comparing the lengths of fillets deposited in the shop with those designated on the drawings, experience shows that the total measured deposit generally exceeds the total on the drawings. However, an individual fillet may be permissibly either greater or less than designated. For example, in a welding shop, the total fillets on one welded truss aggregated 633⅓ linear inches, as compared with 598 inches specified, and the maximum excess of actual length on an individual deposit over that specified reached a maximum of 50 per cent excess while in the two average fillets deficits of 8.8 per cent and 10.0 per cent occurred respectively. These deficits were not necessarily the fault of the operator, for, on work of this character, it is possible to attribute them to inaccuracy in drawings specifying a length of fillet which is impossible to execute.

Welding and Building Codes

The revision of building codes and the preparation of specifications for welded buildings are highly important matters. Much thought and attention have been given to them and, at the present time, many municipalities and engineers have formally recognized metallic arc welding as worthy of consideration wherever structural steel is to be fabricated either in the shop or in the field. The American Welding Society has published its 1928 Code. Copies of this code may be obtained by addressing the society. The Pacific Coast Building Officials' Conference has included in its "Uniform Building Code," Section 2710, the authority to use arc welding. This code also sets forth the essential elements required by engineers and architects for working out designs of welded joints in steel beams and columns used for building construction. The unit stresses as given in this code are to be changed in the near future to agree with those given in the code of the American Welding Society. Over fifty cities have adopted the code of the Pacific Coast Building Officials' Conference. These

cities are listed in the appendix at the end of this publication; and, as will be gathered from the two lists of cities following in the appendix, it appears that, under reasonable supervision, arc welding is an acceptable means of construction in other localities.

The Pennsylvania legislature has recently passed a new law which permits welded construction to be used in cities of the first class. As this law is applicable only to cities of the first class, municipalities below that grade are free to adopt their own codes. This is a good indication that an increase in the use of arc welding in the construction of buildings is foreseen.

Each month, more and more municipalities are considering the advisability of adopting arc welding on steel construction as a means of obtaining quiet in the vicinities of hospitals, hotels, apartment houses, schools, and office buildings—and, wherever it has been used, much satisfaction is evidenced.

Buildings

Arc welding has already been successfully applied to more than fifty buildings, varying in height from one to eleven stories; to a score of bridges, including railroad bridges which carry freight locomotives; to many steel barges; to ships, in which part or all of the steel is connected by welds rather than by rivets; to hundreds of miles of steel pipes for transporting oil, or conveying water as part of municipal water-supply system.

The tallest welded building erected to date is the Hotel Homestead at Hot Springs, Va. This was fabricated, erected, and electric arc welded by the American Bridge Company. Five hundred tons of steel are contained in the structure, which consists of two six-story wings and a central portion of eleven stories, above which a tower rises to a height of 180 feet above ground level.

The next-highest arc-welded building erected to date is the power plant adjoining the Chal-

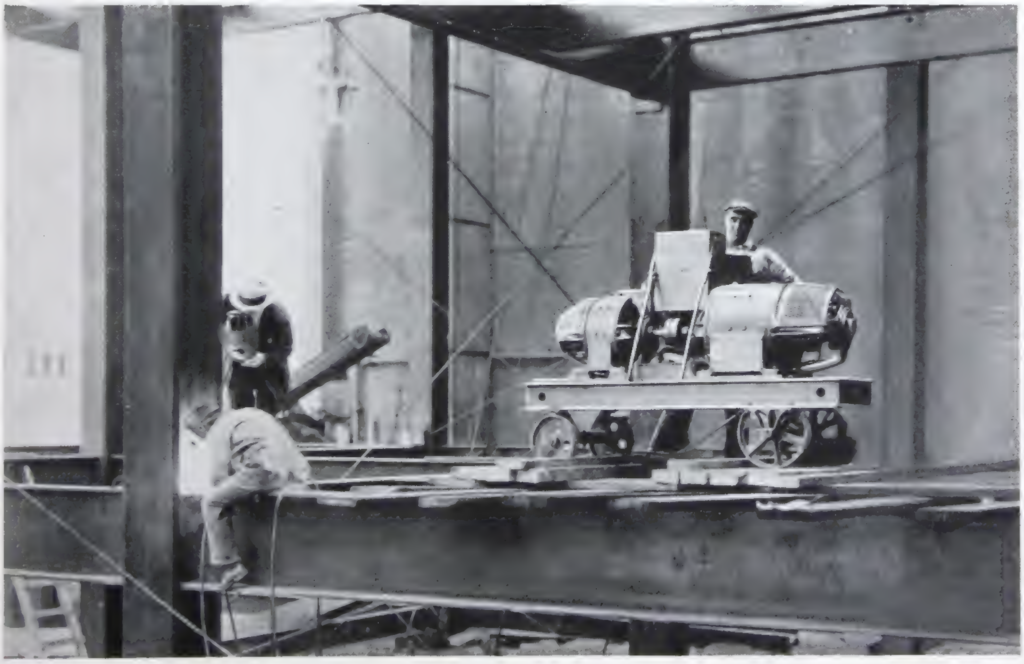


Fig. 24. Welding Six Stories above Ground

fonte and Haddon Hall hotels at Atlantic City, N. J. The steel for this power plant was fabricated, erected, and welded by the Bethlehem Steel Company. It is approximately 150 feet high and contains 540 tons of steel. Steam boilers, coal bunkers, and various other accessories incidental to a power plant of this kind are supported by the framework.

Although the above-mentioned are the tallest welded buildings, they do not contain the greatest weight of steel. The record tonnage to date is contained in the new factory building at the West Philadelphia Works of the General Electric Company, which has approximately 1000 tons of arc-welded steel.

Publication GED-294, which can be obtained from the nearest General Electric sales office, contains a complete list of the various buildings, bridges, locomotives, garages, ships, etc., which have been welded up to the present time.

Fig. 25. Arc-welded Pile Driver, Built to Withstand Severe Shock

Appendix

These Cities Have Adopted, as of June 25, 1929, the Uniform Building Code of the Pacific Coast Building Officials' Conference, Which Includes Arc Welding

Prescott, Arizona
Tucson, Arizona
Yuma, Arizona
Eldorado, Arkansas
Alhambra, California
Alameda, California
Alturas, California
Berkeley, California
Claremont, California
Compton, California
Corcoran, California
Coronado, California
El Monte, California
Fontana, California
Fresno, California
Fullerton, California
Laguna Beach, California
Livermore, California
Madera, California
Martinez, California
Monterey Park, California
National City, California
Oceanside, California
Ontario, California
Oxnard, California
Pasadena, California
Piedmont, California
Pittsburg, California
Pomona, California
Redlands, California
Sacramento, California
San Anselmo, California
San Bernardino, California
San Fernando, California
San Jose, California
San Leandro, California
San Rafael, California
Santa Monica, California
South San Francisco, California
Tujunga, California
Tulare, California
Upland, California
Vixalia, California
Walnut Creek, California
Watsonville, California
Shreveport, Louisiana
Meridian, Mississippi
Grand Forks, North Dakota
Eugene, Oregon
Klamath Falls, Oregon
Salem, Oregon
Ellensburg, Washington
Kelso, Washington

The following is a list of the cities in which it would appear that under reasonable supervision welding is an acceptable tool, because of the fact that welding has been used in them:

Emeryville, California
South Sacramento, California
Bridgeport, Connecticut
New Haven, Connecticut
South Portland, Maine
Pittsfield, Massachusetts
Detroit, Michigan
Atlantic City, New Jersey
Cleveland, Ohio
Youngstown, Ohio
Sharon, Pennsylvania
Providence, Rhode Island
Wheeling, West Virginia

The following are among the principal cities which at present are revising their codes and have welding under consideration:

Birmingham, Alabama
Chicago, Illinois
New York, New York
Niagara Falls, New York
Schenectady, New York
Syracuse, New York
Utica, New York
Portland, Oregon
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Pittsburgh, Pennsylvania
Knoxville, Tennessee
Dallas, Texas
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Charleston, W. Va.	304 Capitol Street
Charlotte, N. C.	200 South Tryon Street
Chattanooga, Tenn.	536 Market Street
Chicago, Ill.	230 South Clark Street
Cincinnati, Ohio	215 West Third Street
Cleveland, Ohio	925 Euclid Avenue
Columbus, Ohio	17 South High Street
Dallas, Tex.	1801 North Lamar Street
Davenport, Iowa	111 East Third Street
Dayton, Ohio	25 North Main Street
Denver, Colo.	650 Seventeenth Street
Des Moines, Iowa	418 West Sixth Avenue
Detroit, Mich.	700 Antoinette Street
Duluth, Minn.	14 West Superior Street
El Paso, Tex.	109 North Oregon Street
Erie, Pa.	10 East Twelfth Street
Fort Wayne, Ind.	1635 Broadway
Fort Worth, Tex.	410 West Seventh Street
Grand Rapids, Mich.	148 Monroe Avenue, Northwest
Hartford, Conn.	18 Asylum Street
Houston, Tex.	1016 Walker Avenue
Indianapolis, Ind.	110 North Illinois Street
Jackson, Mich.	212 Michigan Avenue, West
Jacksonville, Fla.	11 East Forsyth Street
Kansas City, Mo.	1004 Baltimore Avenue
Knoxville, Tenn.	602 South Gay Street
Los Angeles, Calif.	5201 Santa Fe Avenue
Louisville, Ky.	455 South Fourth Street
Memphis, Tenn.	8 North Third Street
Miami, Fla.	25 Southeast Second Avenue
Milwaukee, Wis.	425 East Water Street
Minneapolis, Minn.	107 South Fifth Street
Nashville, Tenn.	234 Third Avenue, North
Newark, N. J.	20 Washington Place
New Haven, Conn.	129 Church Street
New Orleans, La.	837 Gravier Street
New York, N. Y.	120 Broadway
Niagara Falls, N. Y.	201 Falls Street
Oklahoma City, Okla.	15 North Robinson Street
Omaha, Nebr.	409 South Seventeenth Street
Philadelphia, Pa.	1405 Locust Street
Phoenix, Ariz.	11 West Jefferson Street
Pine Bluff, Ark.	501 Main Street
Pittsburgh, Pa.	535 Smithfield Street
Portland, Oreg.	329 Alder Street
Providence, R. I.	76 Westminster Street
Richmond, Va.	700 East Franklin Street
Roanoke, Va.	202 South Jefferson Street
Rochester, N. Y.	89 East Avenue
St. Louis, Mo.	112 North Fourth Street
Salt Lake City, Utah	200 South Main Street
San Antonio, Tex.	201 Villita Street
San Francisco, Calif.	235 Montgomery Street
Schenectady, N. Y.	1 River Road
Seattle, Wash.	821 Second Avenue
Shreveport, La.	513 Marshall Street
Spokane, Wash.	421 Riverside Avenue
Springfield, Ill.	504 East Monroe Street
Springfield, Mass.	1387 Main Street
Syracuse, N. Y.	113 South Salina Street
Tacoma, Wash.	1019 Pacific Avenue
Tampa, Fla.	112 Cass Street
Terre Haute, Ind.	701 Wabash Avenue
Toledo, Ohio	520 Madison Avenue
Trenton, N. J.	143 East State Street
Tulsa, Okla.	409 South Boston Street
Utica, N. Y.	258 Genesee Street
Washington, D. C.	800 Fifteenth Street, Northwest
Waterbury, Conn.	195 Grand Street
Wheeling, W. Va.	40 Fourteenth Street
Worcester, Mass.	340 Main Street
Youngstown, Ohio	16 Central Square

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Detroit, Mich.	700 Antoinette Street
Kansas City, Mo.	819 East Nineteenth Street
Los Angeles, Calif.	5203 Santa Fe Avenue
Minneapolis, Minn.	410 Third Avenue, North
New York, N. Y.	416 West Thirteenth Street
Philadelphia, Pa.	429 North Seventh Street
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ARGENTINA: General Electric, S.A., Buenos Aires, Cordoba, Rosario de Santa Fe, Tucuman, and Mendoza
AUSTRALIA: Australian General Electric Company, Ltd., Sydney, Melbourne, Adelaide, Brisbane, Newcastle, Queensland, Rockhampton, Maffra, Colac, Townsville, Canberra, Albury, and Lismore
BELGIUM AND COLONIES: Societe d'Electricite et de Mecanique (Procedes Thomson-Houston & Carels) Societe Anonyme, Brussels, Belgium
BRAZIL: General Electric, S. A., Rio de Janeiro, Sao Paulo, Bahia, Porto Alegre, Bello Horizonte, Juiz de Fora, Belem, Curityba, Santos, and Recife
CENTRAL AMERICA: International General Electric Co., Inc., Panama City, Panama; Guatemala City, Guatemala; New Orleans, La.
CHILE: International Machinery Company, Santiago, Antofagasta and Valparaiso, Nitrate Agencies, Ltd., Iquique
CHINA: Andersen, Meyer & Company, Ltd., Shanghai; China General Edison Company, Shanghai
COLOMBIA: International General Electric, S. A., Barranquilla, Bogota, Medellin, and Cali
CUBA: General Electric Company of Cuba, Havana, and Santiago de Cuba
ECUADOR: Guayaquil Agencies Co., Guayaquil
EGYPT: British Thomson-Houston Company, Ltd., Cairo
FRANCE AND COLONIES: Compagnie Francaise Thomson-Houston, Paris; International General Electric Co., Inc., Paris; Compagnie Des Lampes, Paris
GERMANY: H. B. Peirce, Representative, General Electric Co., Berlin
GREAT BRITAIN AND IRELAND: International General Electric Co., Inc., British Thomson-Houston Co., Ltd., London, W.C.2; British Thomson-Houston Co., Ltd., Rugby
GREECE AND COLONIES: Compagnie Francaise Thomson-Houston, Paris, France
HOLLAND: Mijnssen & Co., Amsterdam
INDIA: International General Electric Company, Inc., Calcutta, Bombay and Bangalore
ITALY AND COLONIES: Compagnia Generale Di Elettricita, Milan
JAPAN: Shibaura Engineering Works, Tokyo; Tokyo Electric Company, Ltd., Kawasaki, Kanagawa-Ken; International General Electric Co., Inc., Tokyo
JAVA: International General Electric Co., Inc., Soerabaia
MEXICO: General Electric, S. A., City of Mexico, Guadalajara, Monterrey, Vera Cruz and El Paso, Texas
NEW ZEALAND: National Electrical and Engineering Company, Ltd., Auckland, Dunedin, Christchurch and Wellington
PARAGUAY: General Electric, S. A., Buenos Aires, Argentina
PERU: W. R. Grace & Company, Lima
PHILIPPINE ISLANDS: Pacific Commercial Company, Manila; International General Electric Co., Inc., Manila
PORTO RICO: International General Electric Company of Porto Rico, San Juan
PORTUGAL AND COLONIES: Sociedade Iberica de Construcoes Electricas Lda., Lisbon
SOUTH AFRICA: South African General Electric Company, Ltd., Johannesburg, Capetown, Durban, and Port Elizabeth
SPAIN AND COLONIES: Sociedad Iberica de Construcciones Electricas, Madrid, Barcelona, Bilbao, Valladolid, and Sevilla
SWITZERLAND: Trolliet Freres, Geneva
URUGUAY: General Electric, S. A., Montevideo
VENEZUELA: General Electric, S. A., Caracas and Maracaibo

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